State of California

Business, Transportation and Housing Agency

MEMORANDUM

To: Majid Kharrati
Attention: Arturo Jacobo
Office of Design
MS 35

Date: April 12, 2001

File: 11-SD-5/805
KP 48.9 - 51.2 / 43.9 – 46.5
EA 11-0301U1

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICES
Office of Geotechnical Services, MS#63

Subject: Interstates 5 and 805 Interchange, Proposed Retaining Walls.

INTRODUCTION

Following your request, we conducted a geotechnical investigation for the design of several retaining walls proposed along the subject Interchange of Interstates 5and 805. This interchange is to be improved. According to your request, the improvement will involve several auxiliary lanes and new ramps and bridges that will require the construction of several retaining walls. Our investigation consisted of a site reconnaissance, office review of the existing as built plans and geologic maps, geologic mapping, subsurface investigation, engineering analysis, and the writing of this report. Along with your request, we have received from you the proposed structures layout sheets (in scale of 1:500) and cross sections that were used in our fieldwork, engineering analyses, and for report purposes.

PROJECT LOCATION

For the project location and its limits, reference is directed to Figure 1, Project Location. The project site is located in San Diego County, San Diego, California. Along Interstate 805 (I-805) it starts from about 0.9 km north of the Sorrento Valley Boulevard Undercrossing (Station 456+30) and continues north to the merge with Interstate 5 (I-5). Along I-5 it begins at about 0.5 km north of the Genesee Avenue Overcrossing (Station 512+20), continues through the merge with I-805, and ends about 0.7 km south of the Del Mar Heights Road Overcrossing (Station 574+14).

GENERAL

Table 1 on the following page lists the proposed Type 1 retaining walls and indicates their stations limits and maximum heights. For detailed locations of the proposed structures, reference is directed to Figures 2-3A through 2-569.

Table 1

WALL NUMBER	BEGIN STATION	END STATION	STATION LINE	MAXIMUM HEIGHT (m)
3A	2+38.5	2+71.3	CMR	4.42
6	2+00.1	2+09	CMR	1.84
456	456+33	465+22.8	A	5.45
466	466+34	466+89	NT805	3.05
470	470+47	471+68.5	NT805	9.73
512	512+20	514+07	SD	6.91
516	515+13.5	493+37	R-2M	3.35
527	526+91.6	529+60	SB	4.33
543B	540+85	544+02.1	SB	4.20
543A	10+00	12+23.4	Begin: CM3 End: CMR	4.31
544	543+00	545+00	NB	2.77
545A	544+61	546+60	CM4	4.05
545B	10+00	546+38.6	Begin:545B End: CM4	5.62
545C	544+66.1	457+18	SB	4.80
546	545+56.1	546+50	NB	2.79
565	564+10	566+36.9	SD	1.75
569	567+20	574+13.6	SD	6.46

GEOLOGY

The project site lies within the coastal plain section of the Peninsular Ranges Geomorphic Province of California. The coastal plain generally consists of subdued landforms underlain by Cenozoic sedimentary formations.

The project area is generally underlain by two principal rock units: a Mesozoic igneous and metamorphic rock basement and superjacent late Cretaceous, Eocene, Pliocene, Pleistocene, and Holocene sedimentary succession of strata. The basement is composed of Upper Jurassic Santiago Peak Volcanics and mid-Cretaceous granitic rocks of the Southern California Batholith. The post-batholith superjacent sedimentary succession includes Upper Cretaceous Rosario Group, Eocene La Jolla and Poway Groups, Pliocene and Pleistocene San Diego, Lindavista, and Bay Point Formations. Holocene is represented by alluvium, slope wash, landslide, and stream, terrace, and beach deposits. In addition, artificially compacted fill was placed in some areas (Kennedy and Peterson, 1975).

The project alignment lies predominately within Soledad Valley, a tidal marsh with a shallow estuary of Penasquitos Lagoon. To the southeast it extends into the Carroll Canyon valley and to the south it ascends into La Jolla mesa. The project area is underlain by the sedimentary succession of strata. The sediments include Delmar Formation, Torrey Sandstone, Ardath Shale, and Scripps Formation of the La Jolla Group. The post-Eocene deposits include Bay Point Formation, alluvium, and artificially compacted fill. The Delmar Formation typically comprises yellowish-green sandy claystone interbedded with gray coarse-grained sandstone. The Torrey Sandstone is composed of arkosic sandstone, which is white to light brown, medium to coarse grained, subangular, and moderately well indurated. The Ardath Shale comprises predominantly of weakly fissile, olive-gray shale with concretion beds. Bedding of the unit dips generally east to southeast at a shallow angle of 3 to 5 degrees. Expansive claystone locally comprises 25 percent of the Ardath Shale and landslides are commonly associated with these areas. The Scripps Formation typically consists of pale yellowish-brown, medium grained sandstone, often silty or clayey, and occasional cobble-conglomerate interbeds. The Bay Point Formation is composed mostly of marine and nonmarine, poorly consolidated, fine and medium grained, pale brown sand and/or sandstone. The marine part of the formation is fossiliferous and consolidated. It interfingers with unfossiliferous sandstone that lies generally above it, more than 30 m but less than 60 m above the sea level. This part of the Bay Point Formation is considered a nonmarine slope wash. Alluvium consists primarily of poorly consolidated stream deposits of silt (often clayey), sand, and cobble-sized particles derived from bedrock sources that lie within or near the area. These materials were derived as slope wash or were transported and deposited by Penasquitos Creek and its tributaries of Carroll Canyon and Carmel Valley drainages. Alluvium soils are generally considered moderately to highly compressible, and, where granular, subject to liquefaction during a major seismic event. Artificial fill consists of compacted earth materials derived from local sources.

SEISMICITY

No known Holocene fault exists within the project area. However, several secondary faults related to the active Rose Canyon Fault Zone were mapped along the project alignment. These faults, Carmel Valley, Torrey Pines, Salk and a few more unnamed, are currently believed to be pre-Holocene, though no direct evidence supports this fact (Kennedy and Peterson, 1975). The Carmel Valley Fault, the largest of them, offsets Pleistocene sediments of the Bay Point Formation and, therefore, is considered potentially active. Thus, a probability of minor surface rupture along this fault exists.

The nearest known active fault is the Rose Canyon Fault Zone believed to be capable of producing an earthquake with a Maximum Credible Magnitude of 7.0 on the Richter scale. It is located about 5.5 km south and west from the project site. The La Nacion Fault is located about 20 kilometers southeast from the southern end of the project limits, and it is capable of producing an earthquake with a Maximum Credible Magnitude of 6.75 on the Richter scale. In addition, the Elsinore Fault lies about 41 km northeast from the project limits; it is capable of producing an earthquake with a Maximum Credible Magnitude of 7.5 on the Richter scale. All three aforementioned faults are believed to be capable of generating a Peak Ground Acceleration of about 0.5 g at the project site (Mulchin and Jones, 1990).

LIQUEFACTION

Liquefaction, a sudden large decrease of shearing resistance of a cohesionless soil, can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose granular soils that are saturated by a relatively shallow groundwater table are most susceptible to liquefaction and dynamic settlement. Liquefaction is generally known to occur in saturated or near-saturated cohesionless materials at depth shallower than about 30 m. Dynamic settlement, however, can occur in both dry and wet sands at greater depths.

Liquefaction potential during a seismic event on the Rose Canyon Fault is the greatest seismic-related threat to the project. It could initiate settlement of fill embankments especially if these embankments were placed on alluvial soils. Liquefaction could also trigger structural failure. Our evaluation of subsurface conditions along the project alignment indicates that at three proposed retaining wall locations, namely 466, 470, and 527 liquefaction could potentially occur. This is because the Holocene and older Quaternary alluvial soils at those sites are predominantly composed of loose to medium dense sand and silty sand. In addition, the ground water table at those locations is relatively shallow. The liquefaction potential along the alignment of Retaining Wall 470 was evaluated by our office in 1997 and determined to be high (Yazdani, 1997). It was then mitigated by a private developer who installed vibro-replacement stone columns (Hoobs and Askew, 2000). The liquefaction potential along the alignment of Retaining Wall 527, was evaluated using the computer program Liquefy2, and was determined to be moderate. The liquefaction potential along the alignment of Retaining Wall 466, which was based on our subsurface investigation data (borings 466-B1, and 466-B2), was determined to be low due to the predominately clayey nature of the subsurface soils at this location.

FIELD INVESTIGATION

Our field surface investigations consisted of, several site inspections, photo-documentation, and limited geologic 1:500-scale mapping at the locations of the proposed retaining walls. It was conducted mainly in conjunction with the subsurface investigation program that was implemented in 1999. The results of field mapping are presented on Figures 2-3A through 2-569. For photographs of the locations of the proposed structures, reference is directed to Attachment 4, Photos.

The subsurface investigation program consisted of advancing several borings, utilizing Mud Rotary and Coring drilling methods, to depths ranging from 2.4 m to 26.0 m below the ground surface. Boring locations are shown on Figures 2-3A through 2-569. In addition, for this project, we utilized several borings from our 1996 exploration program related to the planed improvements along the project alignment. During the drilling, Standard Penetration Tests (SPT) were performed at selected depth intervals. The SPT tests were performed by dropping a 63.5 kg hammer from a height of 0.762 m on the split-spoon sampler. The 36 mm inside diameter sampler was driven a maximum of 45.7 cm into the soils and the number of blows recorded for each 0.152 m consecutive interval. The value of blows per 0.305 m interval recorded in the boring logs represents the accumulated number of blows that were required to drive the sampler through the last 0.305 m interval into the soils. The recorded SPT values are field values and have not been corrected for overburden pressure. Based on correlation relating SPT blow counts to the relative density of cohesionless soil and the consistency of cohesive soil, the appropriate descriptions of relative density and consistency are indicated on the boring logs presented in Attachment 1, Logs of Borings.

Groundwater was encountered in borings 466-B1 and 466-B2 and its level was recorded on the boring logs. In addition, groundwater conditions exists at the locations of the proposed Walls 470 and 527. Seepage and perched water conditions could have existed in some of our borings, but they could not be detected due to the 'wet' drilling method used. However, seepage water was detected in borings 543-B1 and 543-B2. Groundwater conditions where encountered are discussed further as appropriate in later sections of this report.

For the boring locations, reference is directed to Figures 2-3A through 2-569. Our borings were drilled as close as possible to the alignments of the walls. It should be noted, however, that factors such as drill rig accessibility, job safety, and the presence of underground utilities have often precluded us from locating borings at the desired locations along the alignment of the proposed structures.

At locations where the overall stability of the slope to be retained was of concern, we performed slope stability analyses using the STEDwin 2.5 computer program utilizing cross sections of the proposed slope configurations provided by your office. The slope stability analyses are presented in Attachments 2 and 3.

SITE CONSIDERATIONS, SUBSURFACE SOIL CONDITIONS, AND FOUNDATION RECOMENDATIONS

During the construction of the subject freeway intervals in the mid 60's, the native soils above the freeway grade were cut as either a through cut or side (hill) cut, and the areas below the grade were built up as fill embankments. Also, depending on topography, offramps, onramps, approach, and departure embankments were either cut into the existing native soils or built up as fills. Fill materials generally originated from the nearby native soils cuts. During the grading of the freeway, fill materials were placed and compacted to 90 % Relative Compaction in accordance with CTM 216. In addition, at locations where creeks or drainage courses crossed the freeway alignment, culverts were installed, running typically from east to west.

During our geotechnical investigation, we generally encountered five geotechnical units that underlay the alignment of the proposed retaining walls: artificial fill materials, alluvium, Bay Point Formation, Ardath Shale, and Torrey Sandstone. In addition, in Borings 543-896-6, 543-896-7, and 543-896-8 the Delmar Formation was encountered during the exploration conducted in 1996. However, this formation was encountered at a depth of about 22 m below the elevation of the footing of the proposed Wall 543. Thus, the Delmar Formation, from a geotechnical engineering standpoint, is deemed to be irrelevant.

Based on our subsurface and surface investigation, and the review of previous investigation reports that were performed by Caltrans and various consultants (Yazdani, 1995; Woodward-Clyde, 1993; Leighton & Associates, 1992; Ninyo & Moore, 1996 and 1990), we have established generalized soils parameters that were used in our foundation analyses. The foundation analyses were based on procedures outlined in the FHWA manual (Cheney and Chassie, 1993). Table 2 below lists the pertinent geologic units and their geotechnical parameters.Since alluvial soils were deemed to be unsuitable for foundation support, their parameters were not included in Table 2.

Table 2

GEOTECHNICAL UNIT	COHESION (Kpa)	ANGLE OF INTERNAL FRICTION (degree)	MAXIMUM DRY DENSITY (KN/m^3)
Ardath Shale	14.3	28	18.9
Torrey Sandstone	9.6	33	18.9
Bay Point Formation	24.0	38	19.6
Existing Fill	9.6	32	18.9
Structural Fill/Backfill	9.6	36	18.9

It should be noted that the Ardath Shale, at some locations has been mapped as a bedded unit. Its bedding dips generally east and southeast at a shallow angle ranging from 3 to 5 degrees (Kennedy and Peterson, 1975).

Geotechnical parameters for fill materials were arrived at based on the strength tests on remolded samples of the native formations that were most likely used during the construction of the freeway embankments. The relationship of fill materials to those native soils was confirmed by field index tests performed during our subsurface investigating program.

Several culverts cross under the alignment of the proposed retaining walls. The Caltrans Hydraulics unit should evaluate whether the proposed retaining walls will have any impact on these drainage structures.

Retaining Wall 456

Site Considerations

For the location of the proposed Retaining Wall 456 (Table 1) reference is directed to Figure 2-456. Wall 456 will be about 900 m long and 5.45 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 456+33 to 465+22.8, it will parallel northward the eastern shoulder of the northbound I-805. Along this interval the existing cut slope is southwest-facing, about 9.0 m high (maximum), and inclined at 1:2 vertical to horizontal (V:H) (Attachment 4, Photos 1 through 3). It is our understanding that the proposed wall is to retain planned cuts in the aforementioned slope in order to widen the northbound I-805. Several CMP culverts, running from northeast to southwest, cross under the freeway at Stations: 457+10, 459+52, 461+15, 462+85, and 464+60. A high-voltage electrical overhead line crosses the freeway at about Station 457+52.

Subsurface Soil Conditions

Our surface mapping and subsurface investigation (Borings 456-B1 through 456-B8) revealed that the proposed wall alignment is underlain by native soils of the Ardath Shale. This section of northbound I-805 was constructed as a side cut of the southwest-facing slope. The cut resulted in the exposing of the Ardath Shale on the face of a cut slope and roadway section of the freeway. The Ardath Shale unit consists of a laminated, often fissile, siltstone that locally is gravelly or grades to silty sand, claystone, or sandstone. Their relative consistency, based on SPT blow counts, was determined to be hard. In all eight borings, the Ardath Shale soils were encountered to the maximum depth of exploration. The lowest elevation (about 17.0 m) at which native soils were encountered was at Boring 456-B7. At the time of our investigations, no seepage or spring were observed on the face of the cuts.

Foundation Recommendations

From a geotechnical engineering standpoint, the subsurface conditions along the alignment of Wall 456 are suitable for the design and construction of the proposed maximum 5.45 m high Type 1 retaining wall supported on a spread footing. Based on layouts and cross sections of the proposed wall supplied by your office, and the results of our subsurface investigations, it is our recommendation that the Standard Plan Retaining Wall Type 1 design be used for Wall 456.

Retaining Wall 466

Site Considerations

For the location of the proposed Retaining Wall 466, (Table 1) reference is directed to Figure 2-466. Wall 466 will be about 55 m long and 3.05 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 456+34 to 466+89, it will parallel to the northwest the eastern shoulder of northbound I-805. This interval represents an approach embankment to the Sorrento Valley Boulevard undercrossing. The maximum height of the embankment (fill) of about 7.0 m is at the bridge abutment. Along this interval, the northeast-facing embankment slope descends at a general inclination of 1:3 (V:H) (Attachment 4, Photos 4 and 5). It is our understanding that the subject approach embankment is to be widened to accommodate additional lanes. The widening will involve placing of fill on the existing embankment slope and retaining it with the proposed Wall 466. The footing of the new wall is to be embedded in new fill materials. Two CMP culverts cross under the freeway at Stations 466+38 and 466+70.

Subsurface Soil Conditions

Our surface mapping and subsurface investigation (Borings 466-B1 and 466-B2) revealed that the approach embankment consists of fill materials that are underlain by native alluvial soils. It is estimated that along the proposed wall alignment the fill and alluvial interface is at an elevation of about 11.0 m. We did not explore the fill materials. Our surface mapping and a review of pertinent archived data, however, suggest that the approach embankment was built using materials from cuts in the nearby native formations. The alluvium consists of silty and clayey sand, sand, and sandy clay. Based on SPT blow counts, the relative density of granular soils was determined to range from very loose to loose. The relative consistency of cohesive soils was found to range from very soft to stiff. In both borings, alluvium was encountered to the maximum depth of exploration. The lowest elevation (about –5.0 m) at which alluvial soils were encountered was at Boring 466-B2. Groundwater was encountered at an elevation of about 8.9 m. It should be noted that the proposed wall footing will be supported on fill materials that have not yet been placed. Fill materials should be selected such that the shear strength parameters of the compacted fill will conform to the strength parameters indicated in Table 2.

As indicated above, the underlying alluvial soils are partially comprised of silty and clayey sands. Some of these materials are of low relative density and are present below the groundwater table. During a seismic event these materials could potentially liquefy and cause damage to the wall. The risk level of liquefaction, however, is no greater than that for the existing freeway facilities and any improvements located outside the State Right of Way. Extreme liquefaction mitigation measures such as the use of vibro-compaction or selection of an alternate foundation system such as a deep pile foundation are probably unwarranted given the high costs associated with such measures. For example, if vibro-compaction were used to mitigate liquefaction, the costs associated with this technique would be on the order of about seven hundred thousand dollars. An additional consideration is the relatively low height of the wall (3.05 m maximum). In view of these factors, it is our recommendation that the wall be supported on spread footings bearing on well-compacted embankment fill.

Foundation Recommendations

The subject retaining wall site is underlain by fill over highly compressible sandy and clayey alluvial soils. These materials will be subjected to both elastic and time dependant settlements upon application of fill loads. If the wall were constructed before these time dependant settlements were allowed to occur, the wall could be potentially subjected to large total and differential settlements. Total settlements are estimated to be on the order of about 200 mm. Accordingly, we recommend that the embankment fills be placed and the alluvial soils and existing embankment fill be allowed to consolidate prior to the construction of the wall. The placement of fill should be conducted in accordance with the Standard Specifications and include benching into the existing fill slope. This recommendation is depicted in Figure 19, Wall 466: Typical Surcharge Section. We estimate a time interval of 6 to 9 months for 90% consolidation to be completed. This time interval may be expedited by the use of wick drains. If wick drains are installed at 2 m spacing, the time interval could be reduced to about 60 days.

A settlement monitoring program should be implemented in order to determine whether settlements of the embankment have stabilized prior to wall construction. Settlement platforms should be monitored on a weekly basis after installation and discontinued after at least four consecutive readings indicate that settlements have stabilized.

From a geotechnical engineering standpoint, based on the cross sections of the proposed wall supplied by your office, the results of our subsurface investigations, and on the assumption that the embankment fills will be placed prior to wall construction as recommended above, the Standard Plan Retaining Wall Type 1 design may be used for Wall 466. In addition, Caltrans Hydraulics unit should be contacted to evaluate whether our proposed surcharge recommendations and construction of the new embankment slope will have any impact on the existing culverts.

Retaining Wall 470

Site Considerations

For the location of the proposed Retaining Wall 470, (Table 1) reference is directed to Figure 2-470. Wall 470 will be about 121.5 m long and 9.73 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 470+47 to 471+68.5, it will parallel to the north the base of the east-facing fill embankment of the northbound I-5. This interval represents a departure embankment from the Los Penasquitos Creek Bridge. The maximum height of the embankment (fill), of about 13.5 m is at the bridge abutment. The east-facing embankment slope descends at an general inclination of 1:2 vertical to horizontal (V:H) (Attachment 2, Photo 6 and 25). It is our understanding that the subject departure embankment is to be widened to accommodate additional lanes. The widening will involve placing of fill on the existing embankment slope and retaining it with the proposed Wall 470.

In addition, Wall 470 alignment will be located on the western perimeter of Vista Sorrento Parkway that is a part of the Torrey Reserve Heights development. Following an agreement between Caltrans and the private developer, an island of stone columns was placed at the base of the embankment slope towards the east. This measure, along with a surcharge program to be implemented by the developer, was to mitigate liquefaction and preclude excessive settlements of Wall 470 and the road improvements planned by the developer. Originally, Wall 470 was to be supported on a spread footing embedded in well-compacted fill placed over the island of stone columns. While the wall will still be located west of the proposed Vista Sorrento Parkway, its location has been moved to the west such that it will no longer be within the footprint of the stone column grid. This change has made it necessary to support the wall on a pile foundation.

Subsurface Soil Conditions

Our recent surface mapping and subsurface investigations conducted in 1996 (Borings 470-6101, 470-6102, 470-6103, and 470-6104) revealed that the existing embankment consists of fill materials that are underlain by native alluvial soils of Soledad Valley. It is estimated that along the proposed wall alignment the fill and alluvium interface is at an elevation of about 9.0 m. We did not explore the fill materials. Our surface mapping and a review of pertinent archived data, however, suggest that the embankment was built using predominantly granular materials from cuts in the nearby native formations. The alluvium consists of sand, silty and clayey sand, sandy silt, and silty and sandy clay. Based on SPT blow counts, the relative density of granular soils was determined to range from loose to medium dense. The relative consistency of cohesive soils was found to range from firm to stiff. Based on SPT blow counts, the alluvium is underlain by very dense gravels and cobbles within a sandy matrix that, in turn, are underlain by the sedimentary Ardath Shale Formation. Both, the dense gravels and Ardath Shale geologic unit are, from the geotechnical standpoint, considered to be competent bedrock.

The alluvium and bedrock interface, based on logs of borings, slopes gently south along the Wall 470 alignment. At the location of Boring 470-6104 (Station 471+54.5), it was logged at an elevation of about –6.0 m. At the location of Boring 470-6102 (Station 471+27), it was logged at an elevation of about –7.2 m. At the location of Boring 470-6101 (Station 470+70), it was logged at an elevation of about -9.0 m. Groundwater was encountered at an elevation of 7.2 m.

Foundation Recommendations

The proposed retaining wall should be supported on a CIDH pile foundation extending into competent bedrock. Approximate elevations of the bedrock are shown on Figure 21, Wall 470: Cross Section A—A'.

Construction considerations for installation of the CIDH piles include the presence of shallow groundwater and the possibility of caving loose soils. Casing may be required during installation of the piles. Additionally, due to the proximity of the island of stone columns to the wall, some drilling difficulties may be encountered in gravel backfill.

The retaining wall foundation will be subjected to negligible settlements as a result of it being supported on competent bedrock. The embankment behind the wall will, however, be subjected to large settlements (200 mm to 300 mm). Therefore, it is imperative that after construction of the embankment, a waiting period of 6 to 9 months be incorporated into the plans and specifications to allow for the settlements to occur prior to construction of the pavement section, drainage facilities and any other improvements. The embankment should be constructed prior to installation of the pile foundation in order to minimize any downdrag effect on the piles.

Retaining Wall 512

Site Considerations

For the location of the proposed Retaining Wall 512 (Table 1) reference is directed to Figure 2-512. Wall 512 will be about 187 m long and 6.91 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 512+20 to 514+07, it will parallel northward the eastern shoulder of the northbound I-5. Along this interval the existing cut slope is west-facing, about 31.0 m high (maximum), and inclined at 1:1.5 (V:H) (Attachment 2, Photos 7 and 8). It is our understanding that the proposed wall will retain planned cuts in the aforementioned slope in order to widen the northbound I-5.

Subsurface Soil Conditions

Our surface mapping and subsurface investigation (Borings 512-B1 and 512-B2) revealed that the proposed wall alignment is underlain by native soils of the Ardath Shale. This section of the northbound I-805 was constructed as a side cut of the west-facing slope. The cut resulted in the exposing of the Ardath Shale formation on the face of a slope and roadway section of the freeway. The Ardath Shale unit consists of a laminated, often fissile, siltstone, that locally is gravelly, indurated, or grades to claystone. Their relative consistency, based on SPT blow counts, was determined to be hard. In both borings, the Ardath Shale soils were encountered to the maximum depth of exploration. The lowest elevation (about 49.4 m) at which native soils were encountered was at boring 512-B2. At the time of our investigations, no seepage or spring was observed on the face of the cuts.

Foundation Recommendations

From a geotechnical standpoint, the subsurface conditions along the alignment of Wall 512 are suitable for the design and construction of the proposed maximum 6.9 m high Type 1 retaining wall supported on a spread footing. Based on layouts and cross sections of the proposed wall supplied by your office, and the results of our subsurface investigations, it is our recommendation that the Standard Plan Retaining Wall Type 1 design be used for Wall 512.

Retaining Wall 516

Site Considerations

For the location of the proposed Retaining Wall 516 (Table 1) reference is directed to Figure 2-516. Wall 516 will be about 110 m long and 3.35 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 515+13.5 to 493+37 (R-2M Line), it will parallel northward the eastern shoulder of the northbound I-5. Along this interval the existing cut slope is west-facing, about 30.0 m high (maximum), and inclined at 1:1.5 (V:H) (Attachment 2, Photos 9 and 10). It is our understanding that the proposed wall is to retain planned cuts in the aforementioned slope in order to widen northbound I-5.

Subsurface Soil Conditions

Our surface mapping and subsurface investigation (Borings 516-B1 and 516-B2) revealed that the proposed wall alignment is underlain by native soils of the Ardath Shale. This section of the northbound I-5 was constructed as a side cut of the west-facing slope. The cut resulted in the exposing of the Ardath Shale formation on the slope face and roadway section of the freeway. The Ardath Shale unit consists of laminated siltstone that locally grades to claystone. Its relative consistency, based on SPT blow counts, was determined to be hard. In both borings, the Ardath Shale soils were encountered to the maximum depth of exploration. The lowest elevation (about 42.5 m) at which native soils were encountered was at Boring 516-B2. At the time of our investigations, no seepage or spring was observed on the face of the cuts.

Foundation Recommendations

From a geotechnical standpoint, the subsurface conditions along the alignment of Wall 516 are suitable for the design and construction of the proposed maximum 3.35 m high Type 1 retaining wall supported on a spread footing. Based on layouts and cross sections of the proposed wall supplied by your office, and the results of our subsurface investigations, it is our recommendation that the Standard Plan Retaining Wall Type 1 design be used for Wall 516.

Retaining Wall 527

Site Considerations

For the location of the proposed Retaining Wall 527, (Table 1) reference is directed to Figure 2-527. Wall 527 will be about 268.4 m long and 4.33 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 529+60 to 526+91.6, it will parallel to the south the western shoulder of southbound I-5. This interval represents an approach embankment to the Los Penasquitos Creek Bridge. The maximum height of the embankment (fill), of about 9.0 m is at the bridge northern abutment. The west-facing embankment slope descends at an general inclination of 1:2 (V:H) (Attachment 2, Photos 11 and 12). It is our understanding that the subject approach embankment is to be widened to accommodate additional lanes. The widening will involve the placement of fill on the existing embankment slope and alluvial soils to the west of the embankment toe and retaining the fills with the proposed Wall 527. The footing of the new wall is to be embedded in new fill materials. The alignment of the proposed wall lies at about the base of the approach embankment, along the existing concrete brow ditch. A RCP culvert crosses under the freeway at Station 529+10.

Subsurface Soil Conditions

Our surface mapping and subsurface investigations (Borings 527-B1 through 527-B3, drilled in 1999, and Borings 527-61015, 527-6109, and 527-6108 drilled in 1996) revealed that the approach embankment consists of fill materials that are underlain by native alluvial soils. The alluvial soils extend west and south from the base of the embankment and are underlain by the Bay Point Formation. The fill and alluvium interface was mapped at about Station 529+80 where it dips south at a low angle. In addition, at about the same station, the interface between the Bay Point Formation and alluvium was also mapped. This interface dips south at a low angle as well. The fill and alluvium interface was estimated to be at an elevation of about 8.5 m. Fill consists of layers of siltstone, sandy and silty clay, sandy gravel, and sand. The relative density of its granular components, based on SPT blow counts was estimated to be medium dense. The relative consistency of the fill cohesive components, based on the same criterion, was estimated to be firm to hard. The alluvium consists of sandy and silty clay (locally organic), silt, and clayey and silty sand. Based on SPT blow counts, the relative consistency of its cohesive soils was found to range from firm to very stiff. The relative density of granular soils was determined to range from loose to medium dense. In Borings 527-6109 and 527-6108, alluvium was encountered to an elevation of about –11.0 m. Groundwater was encountered at an elevation of about 0.6 m. It should be noted that the proposed wall footing will be supported on fill materials that have not yet been placed. Fill materials (structural backfill) should be selected such that the shear strength parameters of the compacted fill will conform to the strength parameters indicated in Table 2.

As indicated above, the underlying alluvial soils are partially comprised of silty and clayey sands. Some of these materials are of low relative density and are present below the groundwater table. During a seismic event these materials could potentially liquefy and cause damage to the wall. The risk level of liquefaction, however, is no greater that that for the existing freeway facilities and any improvements located outside the State Right of Way. Extreme liquefaction mitigation measures such as the use of vibro-compaction or selection of an alternate foundation system such as a deep pile foundation are probably unwarranted given the high costs associated with such measures. For example, if vibro-compaction were used to mitigate liquefaction, the costs associated with this technique would be on the order of about one million dollars. An additional consideration is the relatively low height of the wall (4.33 m maximum). In view of these factors, it is our recommendation that the wall be supported on spread footings bearing on well-compacted embankment fill.

Foundation Recommendations

The subject retaining wall site is underlain by fill over highly compressible clayey and silty alluvial soils. These materials will be subjected to both elastic and time dependant settlements upon application of fill loads. Total settlements are estimated to be on the order of about 300 mm. If the wall were constructed before these time dependant settlements were allowed to occur, the wall could be potentially subjected to large total and differential settlements. Accordingly, we recommend that the embankment fills be placed and the alluvial soils and existing embankment fill be allowed to consolidate prior to the construction of the wall. The placement of fill should be conducted in accordance with the Standard Specifications and include benching into the existing fill slope. This recommendation is depicted in Figure 20, Wall 527: Typical Surcharge Section. We estimate a time interval of 6 to 9 months for 90% consolidation to be completed. This time interval may be expedited by the use of wick drains. If wick drains are installed at 2 m spacing, the time interval could be reduced to about 60 days.

A settlement monitoring program should be implemented in order to determine whether settlements of the embankment have stabilized prior to wall construction. Settlement platforms should be monitored on a weekly basis after installation and discontinued after at least four consecutive readings indicate that settlements have stabilized.

From the geotechnical engineering standpoint, based on the cross sections of the proposed wall supplied by your office, the results of our subsurface investigations, and the assumption that the embankment fills will be placed prior to wall construction as recommended above, the Standard Plan Retaining Wall Type 1 design may be used for Wall 527. In addition, Caltrans Hydraulics unit should evaluate whether our proposed surcharge recommendations and construction of the new embankment slope will have any impact on the existing culvert.

Retaining Walls 543A and 543B

Site Considerations

For the location of the proposed Retaining Walls 543A and 543B, (Table 1) reference is directed to Figure 2-543. Wall 543B will be about 317 m long and 4.20 m in maximum height. Wall 534A will be about 223.4 m long and 4.31 m in maximum height. Both are proposed to be Type1 standard retaining walls supported on a spread footing foundation. From Station 544+02.1 to 540+85, Wall 543B will parallel southward the west-facing embankment slope of southbound I-5. From Station 544+03 to 541+96 “SD” Line, Wall 543A will parallel southward the aforementioned embankment slope. From Station 544+03 “SD” Line through Station 12+23.4 CMR Line, Wall 543A will run eastward, parallel to Carmel Mountain Road. Along the entire interval, the existing embankment fill slope is about 26.0 m in maximum height, and inclined at 1:2 (V:H) (see Attachment 2, Photos 13 and 14). The alignment of the Wall 543B footing lies at about two-thirds of the height of the existing slope. The alignment of Wall 543A footing lies at about half-height of the slope.

It is our understanding that the existing embankment is to be widened to accommodate additional lanes. The widening will involve placement of fill on the existing embankment slope and retaining it with the proposed Walls 543A and 543B. The footings of the new walls are to be embedded in newly compacted fill materials. Between the walls, a new fill slope is planned, and it will descend at an inclination of 1:2 (V:H) westward from the bottom of the Wall 543B. Two CMP culverts cross laterally under the freeway along the walls alignment: at Stations 543+83 and 540+84.

Subsurface Soil Conditions

Our surface mapping and subsurface investigation was conducted in 1999 (Borings 543-B1 and 543-B2) and 1996 (Borings 543-896-6, 543-896-7, 543-896-8, and 543-996-10). They revealed that the subject section of the road embankment consists of a top layer of fill materials that is underlain by native soils of the Bay Point Formation. This section of southbound I-5 was constructed during side cut-and-fill grading of the west-facing slope. Fill materials underlie the alignments of the proposed Walls 543A and 543B. The fill consists of sand, locally clayey, with gravel and chunks of siltstone. Its relative density, based on SPT blow counts was estimated to be medium dense to dense. Fill materials are underlain by native soils of the Bay Point Formation. They comprise of dense to very dense sands. Their geotechnical engineering strength parameters are presented in Table 2. The thickness of the fill layer along the wall alignment is variable. At Station 543+80, based on Borings 543-B1 and 543-B996-10, the fill and bedrock interface is estimated to be at about elevation 20.6 m. At Station 543+20, based on Borings 543-B2 and 543-896-6, it is estimated to be at about elevation 15.8 m, and at Station 542+70, based on Borings 543-B2 and 543-896-7, at about elevation 13.4 m.

For boring locations, reference is directed to Figure 2-543. It should be noted that the actual locations of the proposed alignments of Walls 543A and 543B were not accessible to conventional subsurface exploration equipment with the exception of Boring 543-896-10. Our borings were drilled as close as possible to both wall alignments: at the crest of the embankment slope and at its toe. Therefore, the subsurface soil conditions presented above were interpolated, based on the referenced borings. Variations from our estimated soil conditions could exist at the alignment of both walls. In particular, the depth to the Bay Point Formation could vary significantly along the actual alignment of the wall. For example, at about Station 542+20, we expect it to be deeper. Seepage was encountered in Borings 543-B1 and 543-B2. It was detected along the fill and native soils interface: in Boring 543-B1 at an elevation of 26.8 m, and in Boring 543-B2 at elevation of 28.8 m.

Foundation Recommendations

We recommend that along the entire length of the alignment of both walls, from the toe to the crest of the slope, the top 1.2 m layer of fill materials be removed and replaced with structural backfill. This structural backfill should be benched into the existing slope in accordance with Caltrans Standard Specifications and compacted to 95% Relative Compaction in accordance with CTM 216. In addition, new foundation fill and wall backfill should be compacted in accordance with the Standard Specifications to 95 % Relative Compaction. The Structural Backfill material, when compacted to 95 % of Relative Compaction should yield strength parameters no less than those shown for fill/structural fill in the attached Table 2.

From a geotechnical engineering standpoint, based on cross sections and layouts of the proposed walls supplied by your office, the results of our subsurface investigations, and the assumption that the upper 1.2 m of the slope face will be replaced with structural backfill prior to construction of the walls, the Standard Plan Design may be used for Walls 543A and 543B.

Slope stability analysis using the STEDwin computer program was performed on the cross section of the proposed embankment that includes both walls. This cross section was provided by your office. The analysis was performed to determine the stability of the new fill slope. For the analysis, we utilized the soil strength parameters presented in Table 2. The calculated Modified Bishop Safety Factor for the proposed new embankment slope is 1.7, which is considered to be acceptable. The slope stability computer output data are presented in Attachment 2, Walls 543A and 543B: Slope Stability Analysis.

Retaining Wall 544

Site Considerations

For the location of the proposed Retaining Wall 544, (Table 1) reference is directed to Figure 2-544. Wall 544 will be about 200 m long and 2.77 in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 543+00 to 545+00, it will parallel to the north the eastern shoulder of the northbound I-5. This interval, from the southern end of the wall to about Station 544+40, was constructed as a side cut. The cut slope ascends at a general inclination of 1:4 (V:H), and its maximum height is about 13.0 m (Attachment 2, Photo 15). From about Station 545+40 to the northern end of the wall, the interval represents an approach embankment to the Carmel Mountain Road Undercrossing. The east-facing embankment (fill) slope descends at a general inclination of 1:2 (V:H) (Attachment 2, Photo 16). It is our understanding that the subject freeway section is to be widened to accommodate widening of the Carmel Mountain Road Undercrossing. The widening will involve undercutting of the existing slope, placement of fill to the east of the proposed wall alignment, and retaining it with Wall 544. A large diameter underground storm water pipe bounds the southern end of the wall.

Subsurface Soil Conditions

Our surface mapping and subsurface investigations (Borings 544B-B1, 544-B2, and 545B-B3) revealed that the alignment of Wall 544 is underlain by fill materials and native soils of the Torrey Sandstone Formation. Starting from about Station 544+40, the fill and native soils interface dips north at an approximate inclination of 1:4 (V:H). In Boring 544-B3 fill was logged to the lowest elevation of about 21.2 m. Fill materials consist of sands mixed with gravel and chunks of siltstone. Their relative density, based on SPT blow counts, was found to be medium dense. From about Station 544+40 to the north, the native soils of the Torrey Sandstone Formation underlie the wall alignment. This formation consists of sands and sandstone that are locally comprised of thin layers of silty clay. The relative density of native soils, based on SPT blow counts, was found to be medium dense. From a geotechnical engineering standpoint, the Torrey Sandstone is considered to be competent bedrock.

Foundation Recommendations

From a geotechnical engineering standpoint, based on layouts and cross sections of the proposed maximum 2.77 m high wall supplied by your office, the results of our subsurface investigations, and the assumption that the wall foundation will be supported on native soils, the Standard Plan Design Type 1 may be used for Wall 546. At Station 544+40, a construction joint should be incorporated into the wall design in order to mitigate the potential for differential settlement at the fill and native soils interface.

Retaining Wall 545A

Site Considerations

For the location of the proposed Retaining Wall 545A, (Table 1) reference is directed to Figure 2-545A. Wall 545A will be about 99 m long and 4.00 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 546+60 to 544+61, it will parallel to the south the western shoulder of the southbound I-5. The Wall 545A alignment lies at about the base of a slope that descends to the west at a general inclination of 1:2 vertical to horizontal (V:H). The maximum height of the slope is about 8.0 m (Attachment 2, Photos 17 and 18). It is our understanding that the subject freeway section is to be widened to accommodate an offramp to Carmel Mountain Road. The widening will involve undercutting of the existing slope and retaining it with Wall 545A.

Subsurface Soil Conditions

Our surface mapping and subsurface investigations (Borings 545A-B1 through 545A-B3) revealed that the alignment of Wall 545A is underlain by native soils of the Bay Point Formation that in turn are underlain by the Ardath Shale Formation. In Boring 545A-B2, about a 2.3 m thick layer/pocket of compacted fill was encountered. The presence of fill is most likely related to the recent (1999) development project that was completed just west of the State Right of Way. Fill materials consist of sands mixed with gravel and occasional cobbles. The native soils of the Bay Point Formation consist predominantly of sands and sandstone that locally could be silty, clayey, or indurated. From a geotechnical engineering standpoint, both the Bay Point and Ardath Shale geologic units are considered to be competent bedrock. However, during our drilling program we found that the top layer of the Bay Point Formation was intensely weathered, and thus not meeting the competency criterion for bedrock.

Foundation Recommendations

From a geotechnical engineering standpoint, the subsurface conditions along the alignment of Wall 545A are suitable for the design and construction of the proposed maximum 4.05 m high Type 1 wall supported on a spread footing. Based on layouts and cross sections of the proposed wall supplied by your office, and the results of our subsurface investigations, it is our recommendation that the Standard Plan Retaining Wall Type 1 design be used for Wall 545A.

Retaining Wall 3A

For the location of the proposed Retaining Wall 3A, (Table 1) reference is directed to Figure 2-3A. Wall 3A will be about 32.7 m long and 4.43 m in maximum height. It is proposed to be a Type 1 standard retaining wall supported on a spread footing foundation. From Station 2+71.3 to 2+38.6, it will parallel Carmel Mountain Road to the west. It will retain a cut in fill materials and native soils of the approach embankment of the Carmel Mountain Undercrossing. Based on Borings 3A-B1, 3A-B2, and 545B-B5 (Attachment 1), we anticipate that the alignment of Wall 3A is underlain by native soils of the Bay Point Formation. Subsurface conditions along the alignment of Wall 3A are essentially similar to those encountered at the location of Wall 545B. Therefore, based on layouts and cross sections of the proposed wall supplied by your office and the results of our geotechnical investigations, it is our recommendation that the Standard Plan Retaining Wall design be used for Wall 3A.

Retaining Wall 6

For the location of the proposed Retaining Wall 6, (Table 1) reference is directed to Figure 2-6. Wall 6 will be about 10 m long and 1.84 m in maximum height. It is proposed to be a masonry wall. From Station 544+28.3 "CM4" Line to 2+00.2 "CMR" Line, it will generally parallel to the west Carmel Mountain Road. Subsurface conditions along the alignment of Wall 6 are essentially similar to those encountered at the location of Wall 545B (Attachment 1, Boring 6-B1). Therefore, based on layouts and cross sections of the proposed wall supplied by your office and the results of our geotechnical investigations, it is our recommendation that the Standard Plan Retaining Wall design be used for Wall 6.

Retaining Walls 545B and 545C

Site Considerations

For the location of the proposed Retaining Walls 545B and 545C (Table 1), reference is directed to Figure 2-545B. Wall 545B will be about 226.6 m long and 4.7 m in maximum height. Wall 545C will be about 254.4m long and 4.8 m in maximum height. Both walls are proposed to be Type1 standard retaining walls supported on spread footing foundations. From Station 10+00 RW "545B" to about 544+53 "CM4", Wall 545B will run at the base of the embankment slope paralleling Carmel Mountain Road to the west. From about Station 544+53 through 546+38.6, Wall 545B will parallel southbound I-5 in a southward direction. From Station 544+66.1 through 547+18, Wall 545C will parallel the western shoulder of the southbound I-5 in a southward direction. This interval of the freeway, from the northern end of Wall 545C to about Station 545+65, was constructed as a side cut. The cut slope ascends at a general inclination of 1:2 vertical to horizontal (V:H), and its maximum height is about 6.0 m (Attachment 2, Photo 19). From about Station 545+65 to the southern end of Wall 545B, the interval represents an approach embankment to the Carmel Mountain Road Undercrossing. The west-facing embankment (fill) slope descends at a general inclination of 1:2 (V:H) (Attachment 2, Photo 20). It is our understanding that the subject section is to be widened to accommodate an offramp to Carmel Mountain Road and the future widening of the Carmel Mountain Road Undercrossing. From the northern end of Wall 545C to about Station 545+65, the widening will involve undercutting of the existing slope and retaining it with the sections of both walls.

From about Station 545+65 to the southern limit of Wall 545B, the widening will involve placement of fill on the existing embankment slope and retaining it with the section of the proposed upper Wall 545C. Also, throughout this interval a cut at the base of the embankment is to be retained with the proposed lower Wall 545B (see cross section in Attachment 2, Walls 545B and 545C: Slope Stability Analyses).

Subsurface Soil Conditions

Our surface mapping and subsurface investigations (Borings 545B-B1 through 545B-B5) revealed that the alignments of the proposed walls, from their southern limits to Station 545+65, will be underlain by fill materials overlying the native soils of the Bay Point Formation that in turn are underlain by the Ardath Shale Formation. The remaining section of the walls alignments, from about Station 545+65 to their northern ends, is underlain by the Bay Point Formation. Starting from about Station 545+65, the fill and native soils interface dips south at an approximate inclination of 1:7 (V:H). At the location of Boring 545B-B5 the interface was logged at its lowest elevation of about 21 m. Fill materials consist of sands mixed with gravel and occasional cobbles. Their relative density, based on SPT blow counts, ranged from medium dense to dense. The native soils of the Bay Point Formation consist predominantly of sands and sandstone that locally could be silty or indurated. From a geotechnical engineering standpoint, both the Bay Point and Ardath Shale geologic units are considered to be competent bedrock. During our drilling program, however, we found that the top layer of the Bay Point Formation was intensely weathered, and thus not meeting the competency criterion for bedrock.

Foundation Recommendations

We recommend that along the proposed Wall 545C alignment, from Station 545+ 60 to its southern limit, the layer of fill materials be removed and replaced with structural backfill as shown on the sketches designated as Wall 545C: Typical Foundation Improvement Section (Figures 22, and 22A). Since the existing embankment fill wedge under the alignment of the proposed Wall 545C is not uniform, the volume of material to be removed gradually increases from Station 545 +60 (where minimal removal is required) through the southern end of the wall. The structural backfill should be benched into the existing slope in accordance with Caltrans Standard Specifications and compacted to 95% Relative Compaction in accordance with CTM 216. In addition, wall backfill should be compacted in accordance with the Standard Specifications to 95 % Relative Compaction. The Structural Backfill material, when compacted to 95 % of Relative Compaction should yield strength parameters no less than those shown for fill/backfill in the attached Table 2. To mitigate the potential for differential settlement at the fill and native soils interface, a construction joint should be incorporated into the design of Wall 545C.

From a geotechnical engineering standpoint, based on cross sections and layouts of the proposed walls supplied by your office, the results of our subsurface investigations, and the assumption that the removal and replacement of fill materials under the section of the Wall 545C foundation will be implemented prior to construction of Wall 545C, the Standard Plan Design may be used for Walls 545B and 545C.

Slope stability analysis using the STEDwin computer program was performed on the cross section of the proposed embankment that includes both walls. This cross section was provided by your office. The analysis was performed to determine the stability of the new fill slope. For the analysis, we utilized the soil strength parameters presented in Table 2. In addition we assumed that the slope will descend from the base of the Wall 545C to the top of the Wall 545B at an inclination no steeper than 1:1.75 (H:V). The calculated Modified Bishop Safety Factor for the proposed new embankment slope is 2.0, which is considered to be acceptable. The slope stability computer output data are presented in Attachment 2, Walls 545B and 545C: Slope Stability Analysis.

Retaining Wall 546

Site Considerations

For the location of the proposed Retaining Wall 546, (Table 1) reference is directed to Figure 2-546. Wall 546 will be about 93.9 m long and 2.8 m in maximum height. It is proposed to be a Type1 standard retaining wall supported on a spread footing foundation. From Station 545+56.1 to 546+50, it will parallel to the north the eastern shoulder of the northbound I-5. This freeway interval was constructed as a side cut. Recently, this interval was widened to include a 6.0 m wide asphalt concrete paved zone that is flanked to the east by a cut slope inclined at 1:1.5 (V:H). To the east this slope transitions into a 6.0 m wide horizontal (cut) bench that is flanked to the east by a cut slope inclined at 1:2 (V:H) (Attachment 2, Photos 21 and 22). It is our understanding that the subject freeway section is to be widened to accommodate additional lanes.

Subsurface Soil Conditions

Our surface mapping and subsurface investigations (Borings 546-B1 and 546-B2) revealed that the alignment of the proposed Wall 546 is underlain by native soils of the Torrey Sandstone Formation. This formation consist predominantly of sands and sandstone that locally could be silty or/and indurated. Based on SPT blow counts, the relative density of those sands was determined to be very dense. From a geotechnical engineering standpoint, this geologic unit is considered to be competent bedrock.

Foundation Recommendations

It is our understanding that the footing of the proposed Wall 546 will be constructed on a fill layer to be built to raise freeway grade. Therefore, the wall foundation fill pad should consist of structural backfill materials compacted to 95 % Relative Compaction in accordance with CTM 216. The minimum thickness of the foundation pad should be 1.5 m, and its horizontal limits should extend a minimum 1.5 m beyond the edges of the proposed wall footing.

From a geotechnical engineering standpoint, based on layouts and cross sections of the proposed maximum 2.8 m high wall supplied by your office, the results of our subsurface investigations, and the assumption that the wall foundation pad will be constructed as recommended above, the Standard Plan Design Type 1 may be used for Wall 546.

Retaining Wall 565

Site Considerations

For the location of the proposed retaining wall 565, (Table 1) reference is directed to Figure 2-565. Wall 565 will be about 226.9 m long and 1.8 m in maximum height. It is proposed to be a Type 1 standard retaining wall. From Station 566+36.9 to 564+10, it will parallel southward the western shoulder of the southbound I-5. Along this interval the existing cut slope is east-facing, about 12.0 m high (maximum), and inclined at 1:2 vertical to horizontal. A brow ditch parallels the toe of the slope. It is our understanding that the proposed wall is to retain planned cuts in the aforementioned slope in order to widen southbound I-5.

Subsurface Soil Conditions

Our surface mapping and subsurface investigations (Borings 565-B1 and 569-B2) revealed that the proposed wall alignment is underlain by a shallow uncontrolled fill layer that in turn is underlain by native soils of the Bay Point Formation. The interface between fill materials and native soils was mapped at about an elevation of 24.9 m in Boring 565-B1 and 19.8 m in Boring 565-B2. This segment of northbound I-5 was constructed as a cut and fill of the east-facing slope. Uncontrolled fill materials are essentially composed of disturbed Bay Point native soils and consist of sands. Their relative density, based on SPT blow counts, was determined to be very loose to medium dense. The Bay Point Formation consists of locally slightly silty, fine to medium grained sand. The relative density of the Bay Point Formation, based on SPT blow counts, was determined to be loose to medium dense. This lower section of the Bay Point Formation could be considered as slope wash. The lowest elevation (about 15.8 m) at which native soils were encountered was at Boring 565-B2. At the time of our investigations, no seepage or springs were observed on the surface of the cut. It should be noted, however, that landscaped privately owned lots are located at the top of the slope and could contribute to seepage. In addition, the configuration of the cut slope is such that it could create a potential for seepage. We anticipate, however, that the standard drainage detail behind the proposed wall should mitigate the potential for a seepage condition.

Foundation Recommendations

We recommend that along the alignment of Wall 565, the existing undocumented fill materials be removed and recompacted to 95% Relative Compaction in accordance with CTM 216. The anticipated depth of removal is about 1.2 m below the bottom elevation of the footing. The horizontal limits of the removal and recompaction should extend a minimum 1.2 m beyond the edges of the proposed wall footing.

From a geotechnical engineering standpoint, based on layouts and cross sections of the proposed maximum 1.8 m high wall supplied by your office, the results of our subsurface investigations, and the assumption that the subsurface materials along the wall alignment will be recompacted as recommended above, the Standard Plan Design Type 1 may be used for Wall 565.

Retaining Wall 569

Site Considerations

For the location of the proposed Retaining Wall 569 (Table 1) reference is directed to Figure 2-569. Wall 569 will be about 693.6 m long and 6.46 m in maximum height. It is proposed to be a Type1 standard retaining wall. From Station 567+20 to 574+13.6, it will parallel southward the western shoulder of the southbound I-5. Along this interval the existing cut slope is east-facing, about 17.0 m high (maximum), and inclined at 1:2 (V:H) (Attachment 2, Photos 23 and 24). It is our understanding that the proposed wall is to retain planned cuts in the aforementioned slope in order to widen the southbound I-5.

Subsurface Soil Conditions

Our surface mapping and subsurface investigation (Borings 569-B1 through 569-B6) revealed that the proposed wall alignment is underlain predominately by native soils of the Torrey Sandstone and Bay Point Formation. In addition, about a 75 m long southern end of the wall alignment is approximately located at the base of the relatively shallow (about 2.0 m thick) fill embankment. The interface between embankment fill and native soils was mapped at about Station 566+00, where it dips south at a low angle. This segment of the northbound I-5 was constructed as a side cut of the east-facing slope. The cut resulted in the exposing of both formations on the cut slope and roadway section of the freeway. The contact between the Torrey Sandstone and Bay Point Formation, based on the available geologic map, was estimated to be at about Station 570+50 (Kennedy and Peterson, 1975). The Bay Point Formation was encountered in Borings 569-B6 and 569-B1. It consists of locally slightly silty fine to medium grained sand. The relative density of the Bay Point Formation, based on SPT blow counts in Boring 569 –B1, was determined to be dense to very dense. The relative density of the lower section of the Bay Point Formation (below the elevation of 30 m), encountered in Boring 569-B6, based on SPT blow counts, was determined to be loose to medium dense. This lower section of the Bay Point Formation could be considered as slope wash. The Torrey Sandstone was encountered in Borings 569-B2 through 569-B5. It consists of medium grained sandstone or sand with traces of fine gravel. Its relative density, based on SPT blow counts, was determined to be very dense. In all borings, the native soils were encountered to the maximum depth of exploration. The lowest elevation (about 23.5 m) at which native soils were encountered was at Boring 569-B6. At the time of our investigations, no seepage or spring was observed on the surface of the cut. However, it should be noted that landscaped private-owned lots are located at the top of the slope. In addition, the configuration of the cut slope, in conjunction with the variably permeable nature of the native soils exposed in the cuts creates a potential for seepage conditions. In the past, our office provided recommendations to mitigate the seepage water condition that occurred north from the northern limit of the proposed Wall 569. The standard drainage detail behind the proposed wall should, however, alleviate the potential for a seepage condition.

Foundation Recommendations

From a geotechnical engineering standpoint, the subsurface conditions along the alignment of Wall 569 are suitable for the design and construction of the proposed maximum 6.46 m high Type 1 retaining wall supported on a spread footing. Based on layouts and cross sections of the proposed wall supplied by your office, and the results of our geotechnical investigations, it is our recommendation that the Standard Plan Retaining Wall Type 1 design be used for Wall 569.

If you have any questions or comments regarding this report, please call Jeff Tesar at (858) 467-2716 (Calnet 734-4062) or Zia Yazdani at (858) 467-4054 (Calnet 734-4054).

J. Tesar
Jeff Tesar
Associate Engineering Geologist

Zia Yazdani
Zia Yazdani
Associate Materials and Reserarch Engineer

Geotechnical Section 11

REGISTERED GEOLOGIST
JEFF J. TESAR
No. 2137
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
ZIAULLAH YAZDANI
No. 2119
Exp. 3-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

FIGURES

1. Figure 1, Project Location
2. Figure 2-3A, Wall 3A
3. Figure 2-6, Wall 6
4. Figure 2-456, Wall 456
5. Figure 2-466, Wall 466
6. Figure 2-470, Wall 470
7. Figure 2-512, Wall 512
8. Figure 2-516, Wall 516
9. Figure 2-527, Wall 527
10. Figure 2-543, Wall 543A and 543B
11. Figure 2-544, Wall 544
12. Figure 2-545A, Wall 545A
13. Figure 2-545B, Wall 545B and 545C
14. Figure 2-546, Wall 546
15. Figure 2-565, Wall 565
16. Figure 2-569, Wall 569
17. Figure 3, Wall 466: Typical Surcharge Section
18. Figure 4, Wall 527: Typical Surcharge Section
19. Figure 5, Wall 470: Cross Section A—A'
20. Figure 6, Wall 545C: Typical Foundation Improvement Section
21. Figure 6A, Wall 545C: Typical Foundation Improvement Section

ATTACHMENTS

1. Attachment 1, Logs of Borings
2. Attachment 2, Walls 543A and 543B, Slope Stability Analysis
3. Attachment 3, Walls 545B and 545C, Slope Stability Analyses
4. Attachment 4, Photos

REFERENCES

1. Kennedy and Peterson, Geology of the San Diego Metropolitan Area, California, Del Mar Quadrangle, Bulletin 200, 1975.

2. Chenney and Chaassie, Soils and Foundations, NHI Course No. 13212, 1993.

3. Yazdani, Memorandum to Majid Kharrati, Vista Sorrento Parkway Surcharge Grading Plan, Caltrans, July 10, 1997.

4. Hoobs and Askew, Torrey Hills, Vista Sorrento Parkway, Report of Observation Services

Performed During Installation of Vibro-Replacement Stone Columns, Geocon, 2000.

5. Yazdani, Stability Analysis for 11-SD-5/56, 1995(In house files).

6. Woodward – Clyde Consultants, Geotechnical Investigations for the proposed Torrey View, San Diego, California, 1993.

7. Leighton & Associates, Report for Caltrans No. 4 830339-02.

8. Ninyo & Moore, Report for Caltrans No. 101586-07, 1996.

JT

cc:

GPearce
JEgan
RGES.02